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Geotechnical Design with Apparent Seismic Safety Factors Well-Bellow 1

by

George Gazetas[1], Ioannis Anastasopoulos[2], Evangelia Garini[3]

Abstract

The paper demonstrates that whereas often in seismic geotechnical design it is not realistically feasible to design with ample factor of safety against failure as is done in static design, an "engineering" apparent seismic factor of safety less than 1 does not imply failure. Examples from slope stability and foundation rocking illustrate the concept. It is also shown that in many cases it may be beneficial to under-design the foundation by accepting substantial uplifting and/or full mobilization of bearing capacity failure mechanisms.

1. Factors of safety in geotechnical engineering

In engineering practice the unavoidable uncertainties (in loads, geometry, methods of analysis) and the associated severe risks from failure dictate the use of factors of safety, which by definition are greater than 1. In foundation design ample factors of safety (of the order of 2–3) are imposed on the static loads to avoid bearing capacity failure of shallow and deep foundations.

Historically, in seismic design the factors of safety were somewhat lower (by up to 50%), in view of the small probability of seismic occurrence during the lifetime of the facility. Thus, for foundation bearing capacity, a factor of safety of 2 under seismic conditions was deemed sufficient instead of the traditional 3 under non-seismic loads. In

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view of the un-realistically small levels of seismic acceleration of times past (seismic coefficients of the order of 0.05–0.15 prevailed even in regions of very high seismicity), keeping the factors of safety substantial (e.g., ≈ 2) was a prudent, easily satisfied requirement.

With the advent of the accelerograph, the levels of design acceleration increased significantly; this eventually necessitated the adoption of (explicit) factors of safety close to 1 (see for in-stance EC8-5). It will be argued in this paper that the nature of the seismic factors of safety (F_E) is fundamentally different from the static F_S, and that accepting seismic "engineering" F_E (well) below 1 may even lead to a safer overall structure.

2. Earthquake engineering: the realm of "capacity design"

Structural earthquake engineering has long ago embraced the philosophy of "capacity design". The main idea is to design the various constituent members of a structure in such a way that members crucial for its stability, the columns, are stronger than the less critical members, the beams ; and that the plastification of members should result from exceedance of their moment, not their shear capacity, thus avoiding brittle failures. Hence, against the design motion, flexural yielding is *directed* to take place in beams, dissipating energy without endangering the overall structural safety.

"Capacity Design" for foundations has taken a slightly different turn: the overturning moment to be carried by the supporting below-ground members is increased over the calculated bending moment capacity of the superstructure (by applying an "overstrength" factor of about 1.3–1.5). Thus, the "hidden" safety factor utilized in the strength calculation of the concrete cross section is removed. The aim is to ensure that:

- No plastic "hinging" develop below the ground surface; i.e. piles, caps, footings remain structurally nearly elastic
- No mobilization of bearing capacity failure mechanism takes place.

Therefore, since the subsequently utilized explicit seismic factors of safety are kept just above 1, the F_E would be certainly larger than 1. This approach is imposed on foundation design mainly (but not only) because post-seismic inspection and repair below ground is hardly feasible — unlike the above ground structural damage. The past argument of greater uncertainty with soils is still being invoked but less convincingly.

3. Why is it not always feasible in geotechnical engineering to satisfy $F_S > 1$?

The levels of acceleration recorded in the last 30 years, with huge values of both peak (ground) acceleration [PGA] and response spectral acceleration [SA] impose a heavy load on foundations, even when the accepted inelasticity (ductility) of the superstructure is large. As examples, we just mention that several records of Kobe (1995) and Northridge (1994) had PGA values exceeding 0.80 g and maximum SA exceeding 2.0 g. Even small magnitude events, e.g. the 1986 San Salvador M_S 5.7, produced peak acceleration of 0.75 g with proportionally large SA values at not-too-short periods. Calling for nearly-elastic response of the soil-foundation system is not only an expensive demand, but also one that in some cases could not be possibly satisfied (as for example when retrofitting and old structure to meet current code requirements). And in any case such a demand is incompatible with the design for high inelastic action (ductility) of the superstructure. After all it is the failure of the superstructure that could have the most severe consequences.

4. Under seismic base excitation $F_S < 1$ does not imply failure

The factor of safety (F_S) against any type of failure under static permanent loads, denoted here after as F_S, must be kept above 1 to avoid failure (actually "well" above 1 to cover uncertainties). Under seismic shaking, F_S is a function of time, $F_S(t)$. Hereafter by seismic factor of safety we mean the *apparent* $minF_S(t)$ *with respect to time.* We will call it "engineering" factor of safety, F_E.

$F_E < 1$ does not necessarily signify failure. For two reasons, that relate to the nature of seismic excitation:

(a) seismic loading is **cyclic** (and, in fact, with rapidly alternating cycles as well)

(b) the triggering seismic motion is an imposed oscillatory displacement at the base, i.e., it is a **kinematic** excitation, not an external load on the superstructure.

Thanks to (a), the duration of $F_E < 1$ is limited (usually to tenths of a second) and the ensuing displacements are reversed before they reach the point of no return, due to the load reversal. Thanks to (b), the *actual loads* transmitted from the base upward to the critical-to-fail structure are limited by the *actual capacity* of the base of the structure or of the interface separating this structure from the base. In other words, as will be seen below, it is only the apparent "engineering" factor of safety, F_E, that (momentarily) drops below 1.

The consequence of $F_E < 1$ is a *finite* inelastic (permanent) deformation of the system: rotation, horizontal, vertical displacement of foundations, slippage of retaining walls and slope wedges.

4.1 Newmark's sliding block analogue

In his seminal Rankine lecture, Newmark (1965) proposed that the seismic performance of earth dams and embankments be evaluated in terms of permanent deformations which occur whenever the inertia forces on a potential slide mass are large enough to overcome the frictional resistance at the “failure” surface. He proposed the analogue of a rigid block on inclined plane as a simple way of analytically obtaining approximate estimates of these deformations. Since then, the analogue has seen numerous applications and extensions, three of which are shown in **Figs. 1 & 2**.

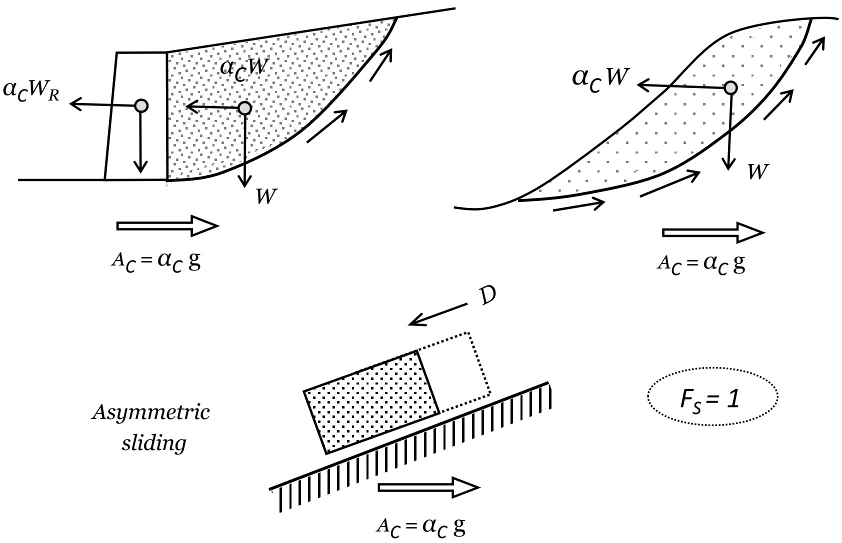

Figure 1. Schematical configurations of geotechnical structures that can be modeled by a rigid block on top of a sloping plane. Definition of critical pseudostatic acceleration.

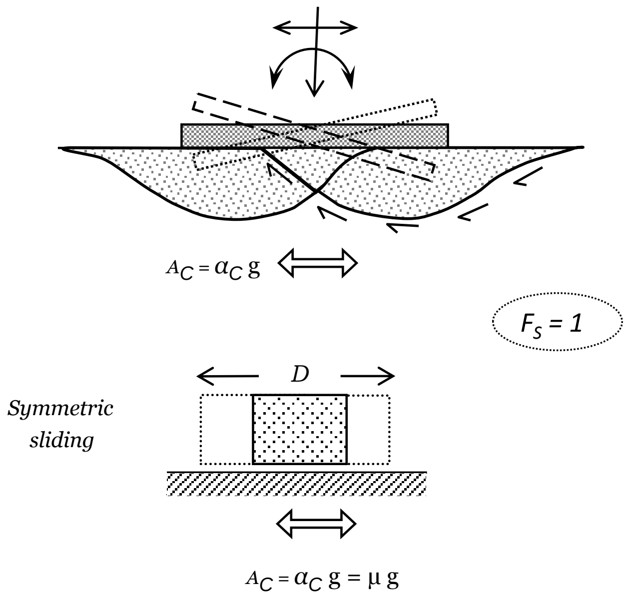

Figure 2. The bearing capacity of a shallow foundation can be modeled by a rigid block on top of a horizontal plane.

The concept of the pseudo-statically deter-mined “critical” or “yield” acceleration, A_C, is a key of the Newmark-type analysis. **Figs 1 & 2** illustrate the concept with two asymmetric and one symmetric geotechnical problems. In the first two, A_C is the pseudo-static “constant” base acceleration which induces inertia forces (*mass* × A_C) in the system that just lead to sliding failure: $F_S = 1$. In the second application A_C is the “constant” base acceleration that induces inertia forces in the superstructure the overturning moment and shear force of which just lead to a bearing capacity failure: $F_S = 1$ (under eccentric and inclined loading). The asymmetric and symmetric sliding block analogues (with an inclined and a horizontal base) are also shown in the two figures.

Newmark (1965) showed that when an embankment or dam is excited by an acceleration of peak amplitude A substantially exceeding the critical acceleration A_C of a prone-to-failure wedge, it will simply experience a permanent (inelastic) downhill displacement — not necessarily excessive so as to constitute failure.

4.2 Examples: slope deformation when F_E <1

Two numerical examples demonstrate the New-mark concept, that an apparent “engineering” factor of safety, F_E , much less than 1 could be accepted in most practical situations as a satisfactory performance.

A slope with $\beta = 25^o$ is sketched in **Fig.3** being 20 m high it consists with a friction angle $\varphi = 36^o$ and is subjected to a base motion in the form of the recently recorded accelerogram,“Lyttelton”, in the M_S 6.3 Christchurch 2011 earthquake. Being very close (not more than 4–5 km) from the seismogenic thrust fault, this record has a substantial peak $A \approx 0.80$ g, along with a large peak velocity of 0.42 m/s. The critical acceleration, for the yield surface shown in the figure, determined by a static slope stability analysis is $A_C \approx 0.20$ g, a value not far from the infinite-slope approximation:

$$A_C \approx \tan(\varphi - \beta)\ \mathrm{g} \approx 0.194\ \mathrm{g} \quad (1)$$

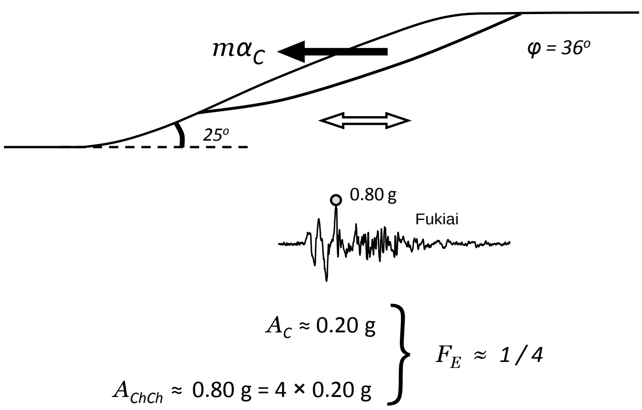

Figure 3. Example of a sandy slope subjected to a strong motion. Apparent engineering factor of safety F_E = 1/4.

Hence, in pseudo-static engineering terms the factor of safety with the chosen excitation is only:

$$F_E = A_C / A \approx 1/4 \quad (2)$$

Now, let us perform a dynamic analysis employing the Newmark analogue : an inclined base of $\beta = 25^o$ and a coefficient of friction, μ, between block and base such that downward sliding is initiated by an upward "pseudo-static" acceleration parallel to the base and equal to:

$$A_C = (\mu \cos\beta - \sin\beta)\ \mathrm{g} = 0.20\ \mathrm{g} \quad (3)$$

from which : $\mu \approx 0.7$. The results of the analysis are graphically illustrated in **Fig. 4**. The top two plots superimpose the block response (acceleration and velocity) on the base excitation. It is noted that the two acceleration histories coincide when their direction is leftward (-), since the (opposite) inertial force on the block cannot cause it to slide uphill ––

hence block and base are *one*, moving together. In the other direction of shaking (+), however, the (opposite) inertial force acts downward causing slippage, every time $A > A_C$. Notice that the largest acceleration of the block when sliding is just equal to A_C .

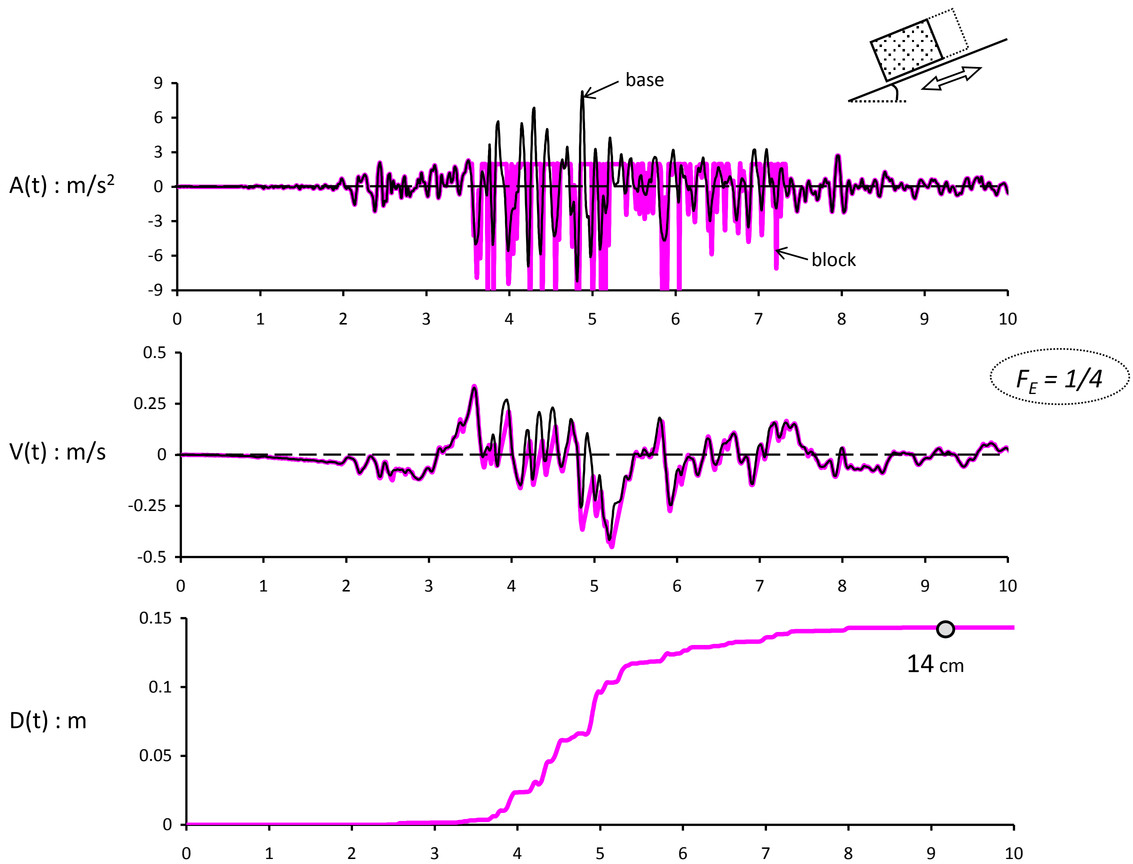

Figure 4. Acceleration, velocity and sliding response of the critical wedge of the slope of Figure 3, modeled with the inclined plane analogue. (Excitation: Lyttelton Port record, 2011 Christchurch EQ, New Zealand).

The consequence is an accumulation of slip-pages which by the end of shaking reach 14 cm. For most slopes and for such a strong shaking, this would be an acceptable displacement.

A second example of a steeper slope, $\beta = 29^o$, of the same material, $\varphi = 36^o$; is subjected to a more typical strong ground motion : the Monastiraki record of the $M_S \approx 6$ Parnitha (Athens) 1999 earthquake. Being 12 km away from the seismogenic fault the record has a peak acceleration $A \approx 0.51$g, but due to its relatively-high frequency content its peak velocity is only 0.15 m/s. As the critical acceleration this time is

$$A_C \approx \tan(36\text{–}29)\ \mathrm{g} \approx 0.13\ \mathrm{g}$$

the apparent engineering factor of safety is again

$$F_E = 0.13 / 0.51 \approx ¼$$

The results of the dynamic analysis are graphically portrayed in **Fig. 5**. The trends are similar to those of the previous example, but due to the shorten duration of each slippage (thanks to the higher excitation frequencies) the final permanent downhill displacement is merely 3.5 cm — hardly a noticeable movement after a strong seismic event.

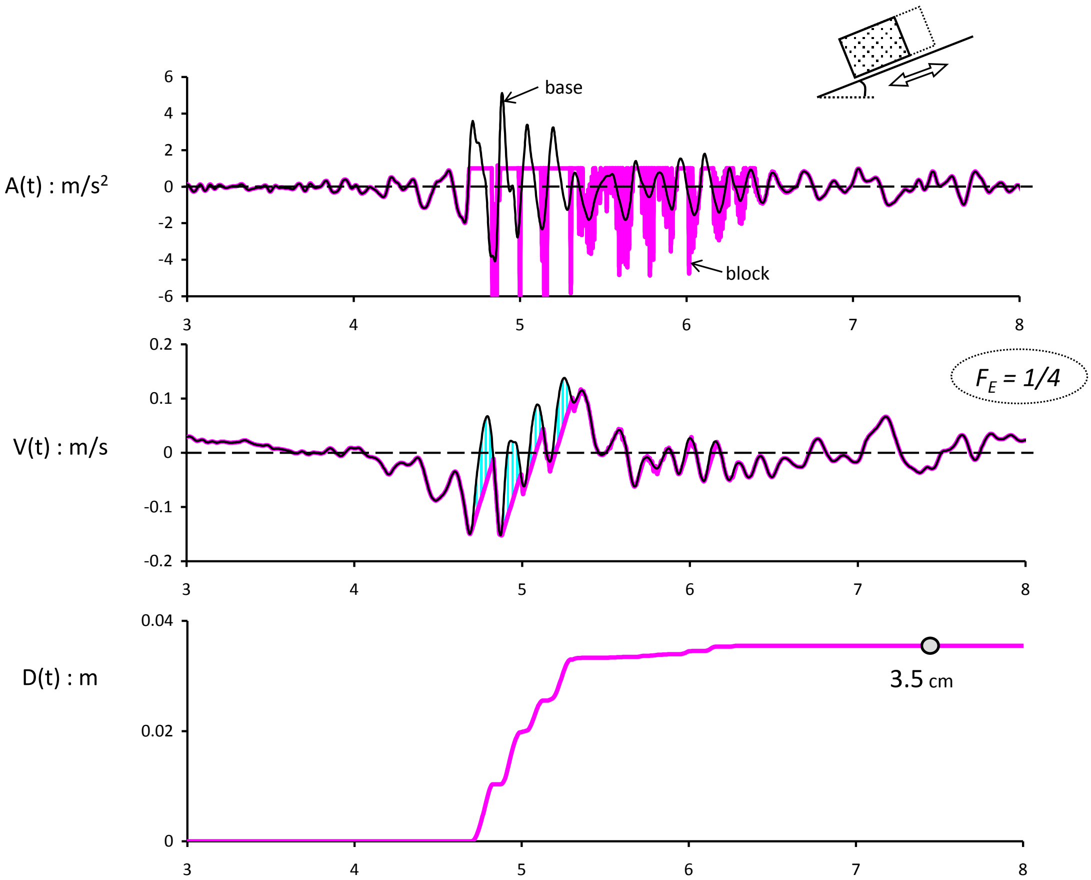

Figure 5. Acceleration, velocity and sliding response of the critical wedge of a β = 29°, ϕ = 36° slope subjected to the Monastiraki record (1999 Parnitha).

We mention (without the proof here) that a 2D finite element analysis of each slope with the accelerograms imposed as horizontal base motion and the material obeying an extended Mohr-Coulomb constitutive law results in even smaller inelastic permanent displacement than the 14 cm and 3.5 cm computed with the Newmark analogue simplification. This further reinforces our main conclusion: $F_E << 1$ does not lead to failure — not always, anyway.

4.3 Rocking and Toppling of Structure on Rock

A first simple proxy of a tall structure forced into rocking motion from a base seismic excitation is sketched in **Fig. 6**: a rigid rectangular block (2b × 2b × 2h) resting a rigid base with tensionless but frictional contact. The pseudo-static critical acceleration A_C of such a block refers to the over-turning of block (in the direction opposite to the constant acceleration). Apparently :

$$A_C = (b / h)\ \mathrm{g} \qquad (4)$$

as explained in **Fig. 6**. Let us now see how the block will behave when excited by accelero-grams with peak $A > A_C$.

As an example a wooden rectangular block 9 × 9 × 30 cm^3 is placed on the Shaking Table of our Laboratory (Drosos et al 2012). Under a constant one-directional (i.e., "pseudo-static") base acceleration just exceeding the critical acceleration $A_C = (9 / 30)\ \mathrm{g} \approx 0.30\ \mathrm{g}$ the block will overturn. Instead we subject it to the so-called Ricker wavelet, an interesting simple motion containing three main peaks of amplitudes: $A = 1.20$ g (the largest) and *0.72* g the other two. Thus, the apparent factor of safety is $F_E = ¼$.

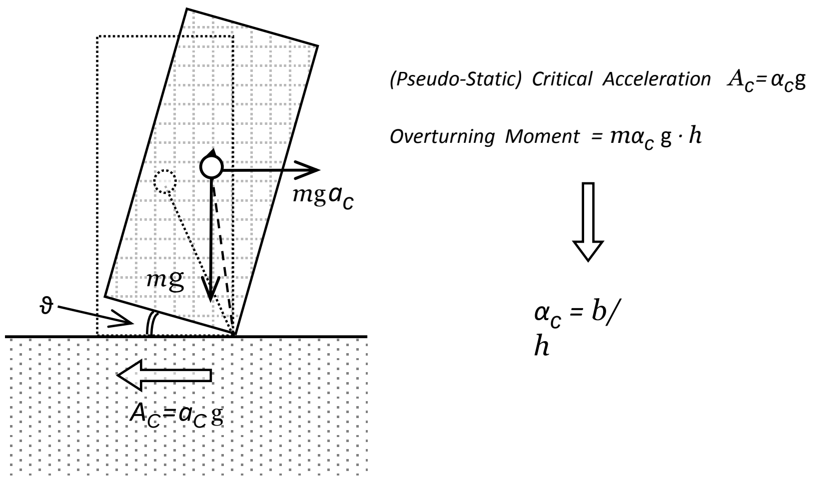

Figure 6. A slender rigid block (width *2b*, height *2h*). Definition of critical pseudo-static acceleration.

Three different dominant frequencies are parametrically chosen for the wavelet: 0.5 Hz, 1 Hz, 4 Hz. The latter two are more representative of usual seismic ground accelerograms. The former is typical of really unique records bearing the effects of near-fault forward-rupture "directivity" and "fling-step" (see Garini et al, 2011). The videos of the three experiments in the laboratory reveal that in none of the three cases do we have toppling of the block (and of course there is no such a thing as a residual rotation —– the system is self-centering).

The recorded time histories of rotation depicted in **Fig. 7** verify the observed survival. The low-frequency wavelet, the most dangerous, produces a maximum angle of rotation of about 0.26 rad, not far from the "overturning" angle

$$\vartheta_C = \arctan (b/h) \approx 0.29 \text{ rad} \qquad (5)$$

The higher frequencies produce much less rotation. The wavelet with *f = 4 Hz* in particular (which frequency is about the mean dominant frequency of most spectral attenuation relations !) is barely uplifting the block, and the only thing one notices in the reality of the *physical* experiment is just a trembling motion. Hence, an engineering F_E much less than 1 does not lead to failure by overtopping of slender rigid structures.

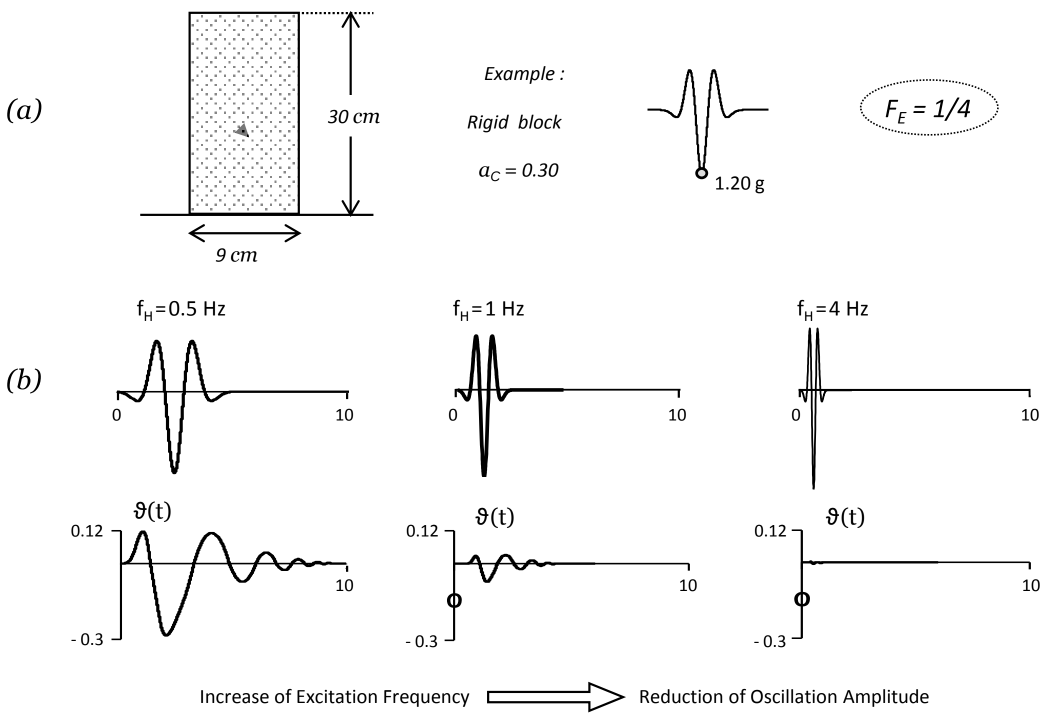

Figure 7. (a) A rectangular rigid block subjected to Ricker excitation; (b) Despite F_E being ¼, the block of Figure 7(a) does not topple. As Ricker pulse frequency increases the rocking response is reduced.

4.4 Rocking and Mobilization of Soil Failure

Avoiding bearing capacity failure under eccentric and inclined load transmitted from the structure onto the foundation has been of great concern to geotechnical engineers. Hence the traditional generous related factors of safety. So, it may come as a great surprise that mobilization of such failure mechanisms under the foundation during seismic shaking does not necessarily lead to failure, but simply to an (additional) permanent settlement and rotation. Depending on the magnitude of such irrecoverable deformations, their development may well be acceptable in many situations.

An example of a simple one-bay five-storey building frame founded with a rigid raft foundation on soft saturated silty soil is presented here (**Fig. 8**) to demonstrate and explain the non-fatal consequences of bearing capacity mobilization under seismic excitation. The definition of critical acceleration is illustrated in the figure. Under a one-directional base "pseudo-static" acceleration, A_C , the inertia forces on each floor lead to an overturning moment *M* and a shear force *Q* on the foundation ; in combination with the vertical load *N*, these static loads lead to a bearing capacity failure with uncontrollable permanent rotation and perhaps toppling of the building (a likely consequence for tall structures in which P-Δ effects could prove devastating).

In the particular example (from a historic significant earthquake) $A_C \approx 0.12$ g. With our understanding of the beneficial role of a high dominant excitation frequency, we deliberately select a low-frequency (hence harmful) motion from the Kocaeli (1999) earthquake. With a peak acceleration $A = 0.36$ g, as base excitation: $F_E = 1/3$.

The results are given in **Fig. 9** in the illuminating form of three snapshots of the response of the structure–soil system at *t = 4 s, 8 s, 17 s.* The last depicts the final stage, at the end of shaking. The first two are at moments when failure mechanisms have developed in the soil under the supporting edge of the foundation: below the left side when *t = 4 s* and below the right side when *t = 8 s*. Evidently, thanks to the alternating (cyclic) nature of the vibration, none of these soil "failures" lasts long. Soon it is being stopped, reversed, and essentially cancelled-out by the "failure" mobilization under the other side. The end result, seen at *t = 17 s*, is mainly a settlement and a (permanent) rotation. These may well be acceptable in many cases.

5. It may even be beneficial to design with $F_e < 1$.

In recent years several researchers have entertained the idea that *"capacity design"* for foundations may be un-necessarily conservative and technically a rather inferior idea (Pecker 1998; Martin & Lam 2000; Kutter et al 2003; Mergos & Kawashima 2005; Harden et al 2006).

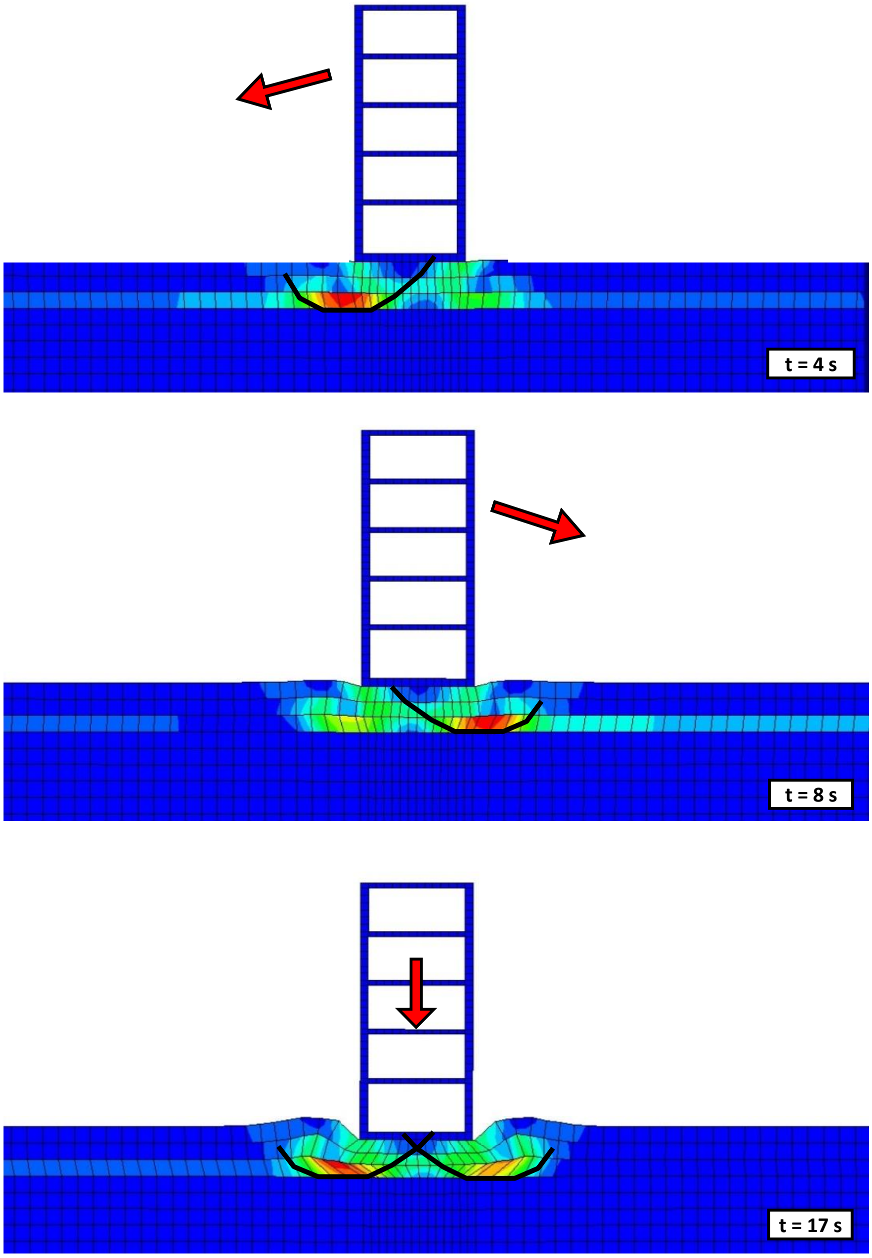

Figure 9. Snapshots of the slender building triggered by a record with A = 0.36 g. Contours of the maximum shear strain are illustrated, revealing the failure zones at every instant.

The author and his coworkers have extended the idea by calling for a reversal of the current capacity design (Anastasopoulos et al 2009; Kourkoulis et al 2012; Gelagoti et al 2011, 2012). Instead of *over-designing* the foundation to ensure that it will not be damaged, we *under-design* it so that it may act as a "safety valve" protecting the superstructure from large accelerations. To this end, the overstrength factor is re-versed to become an *understrength* factor (i.e, we multiply by 0.70 or less rather than 1.40 the structural moments). It is thus hoped that during strong seismic shaking the under-designed foundation will mobilize the inelastic mechanisms in the soil and at the soil-footing interface; such plastic "hinging" below the ground surface will limit the transmitted motion on the superstructure and allow it to perform without plastification.

The concept is demonstrated with the example of **Fig. 10**. A reinforced-concrete bridge pier, with the shown dimensions and deck load, is supported on a stiff clay layer with two different square footings: one, 11 × 11 m^2, conventionally (and conservatively) designed, and the other 7 × 7 m^2 unconventionally (and rather daringly) de-signed in accord with this new philosophy. (The superstructure remains the same.) For a seismic coefficient $C_S = 0.30$ appropriate for design in an EC8 region of the highest seismicity with $A \approx 0.36$ g and a behaviour factor of about 3, the two foundation designs have the following pseudo-static characteristics :

B = 11 m : $F_S = 5.8$, $F_E = 2.0$, $e \approx B/3$

B = 7 m : $F_S = 2.8$, $F_E = 0.5$, $e > B/3$

(Note that for the conventional footing the con-trolling criterion is the magnitude of eccentricity which cannot exceed B/3 — hence the resulting substantial $F_E = 2$. No such limitation is imposed to the unconventional footing.)

We subject the two systems to a severe record, Takatori, from the Kobe 1995 disastrous earth-quake. As its peak ground acceleration is 0.62 g, about two times C_S, the apparent engineering factor of safety against bearing capacity failure of the conventional and unconventional footings are, respectively, $F_E \approx 1$ *and* $F_E \approx 1/4$.

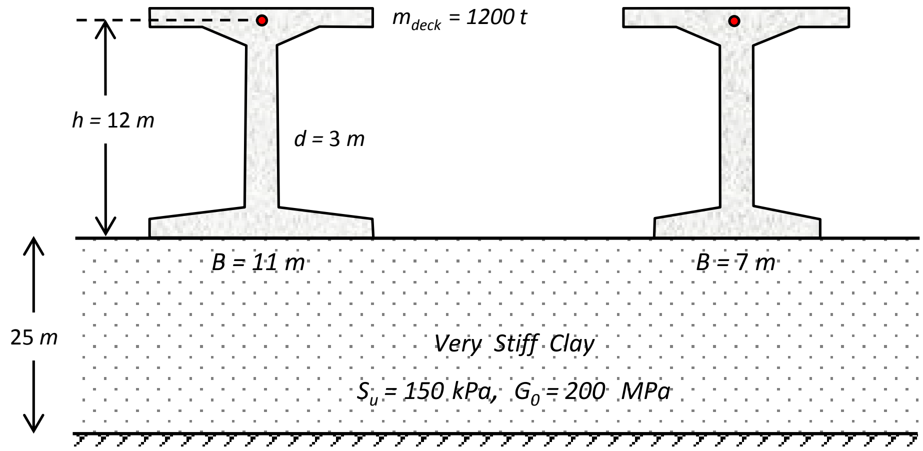

Figure 10. Bridge pier on two different foundations: the conventional 11×11 m^2 and the unconventional 7×7 m^2.

The record and its 5%-damped response spectrum are shown in **Fig. 11**, on which the fundamental periods of the two systems ($T_{B\,=\,11m} \approx 0.70$ s and $T_{B\,=\,7\,m} \approx 1.15$ s) are depicted and re-veal that they correspond to the same spectral acceleration of about 1.5 g. (Hence, the comparison will be quite fair, if not a little disadvantageous for the unconventional system the [anticipated due to inelasticity] lengthening of the period of which will bring it into more severe shaking environment — an ascending response spectral branch.)

Admittedly, shaking with the Takatori record is a very severe testing, far more that the above two apparent factors of safety reveal.

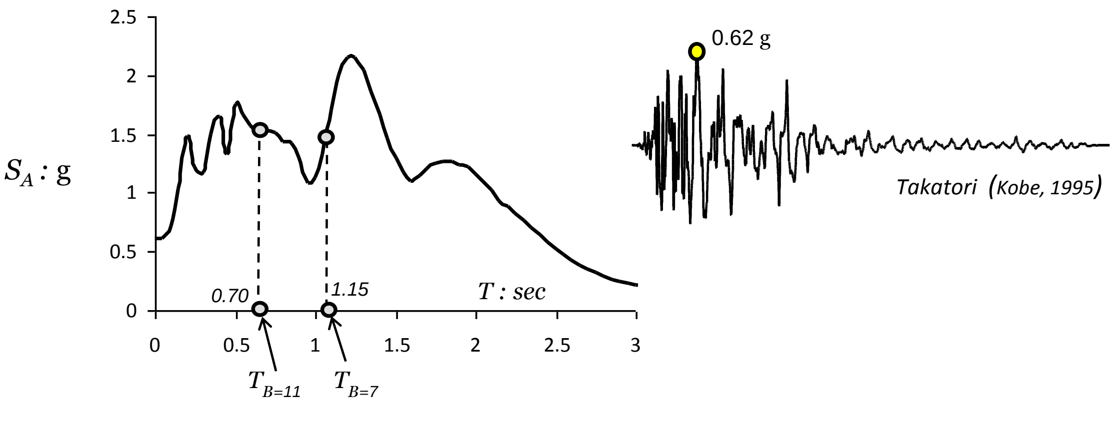

Figure 11. Elastic acceleration response spectrum of the Takatori ground motion.

Fig. 12 vividly shows the consequences of the shaking. The conventional foundation, with its big size, barely induces some inelastic action under the edges of the footing; but the column base develops a plastic hinge with large irrecoverable deformation. Because of its substantial permanent rotation, P-Δ aggravation "pushes" it to collapse.

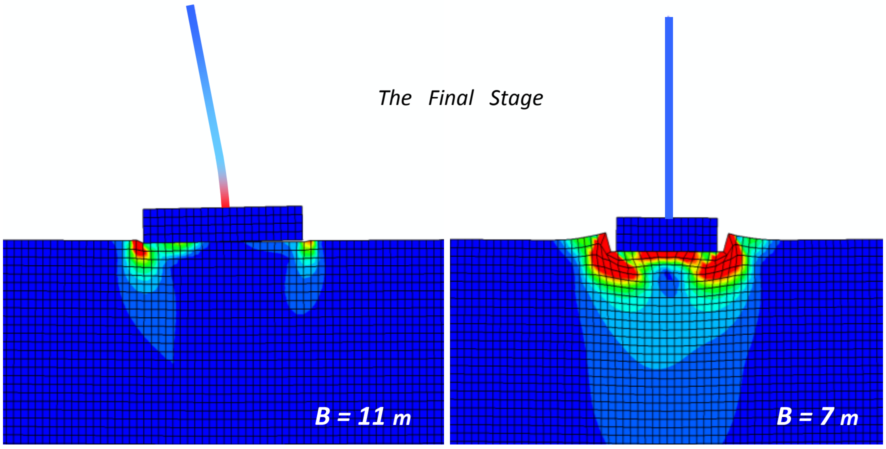

Figure 12. Snapshots of the final stage of the modeled systems triggered by the Takatori record. The conventionally founded ($F_E \approx 1$) pier fails, while the one unconventionally founded ($F_E \approx ¼$) survives but settles.

By contrast, the small footing undergoes large rocking oscillations which produce mobilization of bearing capacity mechanisms, alternating under each side. The end result is a (permanent) settlement of 10 cm with an imperceptible (permanent) rotation of the foundation. But the superstructure remains elastically safe. Whether this settlement is acceptable or not depends, of course, on the type and function of the supported structure. But despite such a small F_E and against such a pernicious earthquake shaking, the unconventional system survived — with injuries, undoubtedly.

6. Conclusions

(a) Pseudo-static factors of safety greater than (or equal to) 1 must not be un-necessarily enforced in earthquake geotechnical engineering, if realistic levels of effective ground acceleration describe the seismic threat.

(b) In foundation design, mobilization of failure mechanisms in the soil or at the soil-footing interface: (i) do not necessarily lead to system failure; and (ii) their development may have a beneficial effect for the supported structure thanks (largely) to the reduction in transmitted accelerations.

(c) A potential price to pay: the residual angle of rotation and settlement may exceed the serviceability limiting values for some sensitive structural systems. Appropriate design improvements may help reduce their magnitude to acceptable levels.

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